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FINITE ELEMENT MODELLING OF THE CYCLIC BEHAVIOUR OF CLT CONNECTORS AND WALLS

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ABSTRACT: The characterization of the behaviour of connectors used in Cross-laminated Timber (CLT) structures is an important aspect that needs to be considered in their seismic design. In this paper, the data from shear and axial tests conducted on connectors have been used to define their force-displacement curves under cyclic loads using the SAWS model in OpenSees. The component curves were then incorporated into the corresponding wall models and the results were compared with their experimental counterparts, in order to determine the validity of the finite element model. Thereby, the non-linear behaviour was restricted to the connectors while the walls themselves were composed of linear orthotropic shell elements. The models were found to provide a good estimate of the initial stiffness and maximum load capacity of the wall specimens. The effects of vertical loading and the presence of openings were determined based on analyses run on the calibrated model.

KEYWORDS: connections, numerical modelling, cross-laminated timber, cyclic tests

1 INTRODUCTION

The scrutiny of the seismic performance of full-scale Cross-laminated timber structures ranging from two to seven storeys [1-3] has demonstrated the self-centring capability of this construction system. Although the results of these experimental campaigns have evinced that CLT is a strong contender for multi-storey building solutions employed in seismic areas, no specific guidelines for the design and construction of these structures exist in the Eurocodes at present. Hence, it is essential to focus on design optimization and assess the impact of structural configuration, connection detailing and layout on the behaviour of these structures.

In general, CLT buildings are erected using the platform construction technique, so the connections between the wall panels of subsequent floors and the wall panels of the ground floor with the foundation need to be designed such that the load paths remain continuous and these structures can rack and slide relative to the foundation, without significant damage to the superstructure [4]. An important part of this process is to determine the strength and deformation capacities of the connectors through experimental tests, which can serve as a basis to develop numerical models that are adequately accurate. The load-slip behaviour of components under monotonic loads is relatively easier to predict whereas, under cyclic loads, the determination of the strength and the deformation capacity is more complex due to the effects of load reversal. In the case of wooden members subjected to cyclic loading, the following phenomena need to be taken into consideration:

- Nonlinear inelastic load-displacement relationship without a distinct yield point.
- Stiffness degradation manifested as a progressive loss of stiffness in each loading cycle.
- Strength degradation manifested as a decrease in strength when cyclically loaded to the same displacement level.
- Pinched hysteresis loops where the loops appear to be pinched in the middle due to the softening of connection joints [5]

Cyclic tests are appropriate for obtaining information on the design level response of a structure, especially in the event of an earthquake. In this paper, the focus will be on understanding and simulating the behaviour of the angle brackets and hold-downs that transfer shear and axial forces between the wall panels and the foundation using finite element models developed in OpenSees [6].

2 EXPERIMENTAL TESTS

As part of an experimental campaign on the study of components of CLT structures, the Graz University of Technology, Institute of Timber Engineering and Wood Technology (TU Graz) in cooperation with the competence centre holz.bau forschungs gmbh (hbf)

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conducted a series of quasi-static tests on single joints and walls with different connection configurations [7,8]. The connectors used in this study are Simpson Strong-Tie angle brackets, AE116, and HTT22 hold-down connectors. The angle brackets had 14 CNA ring shanked nails of 4mm diameter each in the part attached to the wall while two M12 bolts connected the bracket with the base. The metal part of the hold-downs was connected to the CLT wall using 15 CNA ring shanked nails, each with a diameter of 4mm, while the connection with the base was achieved using a single M16 bolt.
Monotonic and cyclic tests were conducted as per the guidelines given in [9] and [10] with a test set-up shown in Figure 1. The cyclic shear tests were performed using a reversed cyclic procedure with predefined values calculated from the monotonic tests previously conducted. For the tension tests, the restricted movement in compression dictated the need for a modification in the load application prodecure. The load-slip behaviour of the angle brackets in shear and tension and of the hold-downs in tension was determined.

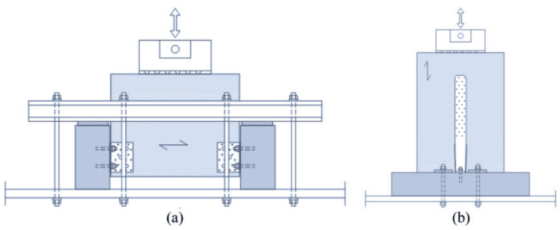

Figure 1: *Test set-up for (a) angle brackets in shear (b) hold-downs in tension [7]*

The subsequent phase of the experimental campaign involved the determination of the load-slip behaviour of several CLT wall panels with different connection configurations, out of which only the two types seen in Figure 2 have been used in this study. While Type A walls used only angle brackets, walls of Type B had both angle brackets and hold-downs.

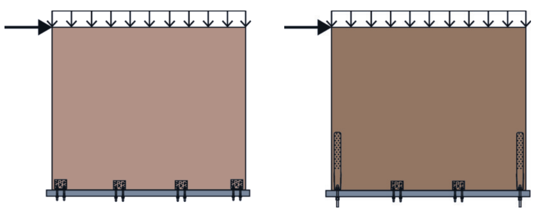

Figure 2: *Wall Type A (left) and Type B (right)*

For walls of type A, two axial loading cases were considered – 0kN/m and 20.8kN/m and for walls of Type B, vertical loads of 5kN/m and 20.8kN/m were applied. The wall panels were square CLT panels of dimensions 2.5m x 2.5m. The wall layup of 40-32-40mm gave rise to a total wall thickness of 112mm. The rigid foundation was simulated by a steel base. All the specimens were subjected to cyclic loading carried out as per the guidelines in [11]. A detailed explanation about the connection behaviour and the performance of the walls during the quasi-static tests can be found in [7].

3 CONNECTOR LEVEL MODELLING

The simulation of the connection behaviour under cyclic loading involved the selection of a suitable hysteresis model from those available in OpenSees. Due to the possibility of allowing for strength and stiffness degradation through the definition of suitable parameters, the SAWS model [12] was chosen. Based on the force-deformation (F-δ) relationship proposed by Folz and Filiatraut [13] for wood shear walls under cyclic loading, the SAWS model in OpenSees requires the user to assign values for ten pre-defined parameters. The degradation parameters, α and β are calculated using equations (1) and (2). Parameters F_0, F_I, R_1, R_2, R_3, R_4, S_0 and Du that control the loading path are determined as shown in Figure 3.

$$\alpha = \frac{\log(K_p/S_0)}{\log\left(\frac{F_0/S_0}{\delta_{max}}\right)} \quad (1)$$

$$\beta = \delta_{\max}/\delta_{\mathrm{un}} \quad (2)$$

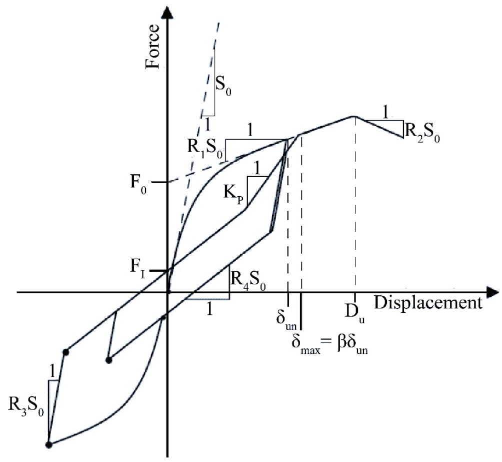

Figure 3: *SAWS model [13, adapted]*

In the first step, the single joint tests were replicated numerically. The angle brackets and hold-downs were defined as zero length elements and assigned with the uniaxial SAWS material property. The ten parameters were determined from the load-slip curves obtained from the experimental tests. In the case of the angle brackets, the characterization of the tensile and shear behaviour was done separately, just as in the single joint tests.
The load-slip curves obtained from the OpenSees models were then compared with the experimental hysteresis curves and presented in Figures 4-6. A clearer picture of the comparison between the experimental and numerical load-slip behaviour was made by listing the magnitude of energy dissipation and maximum load for all three cases in Table 1. At the connector level, it can be seen that the SAWS model can replicate the hysteresis behaviour of the tested connections with a high level of accuracy.

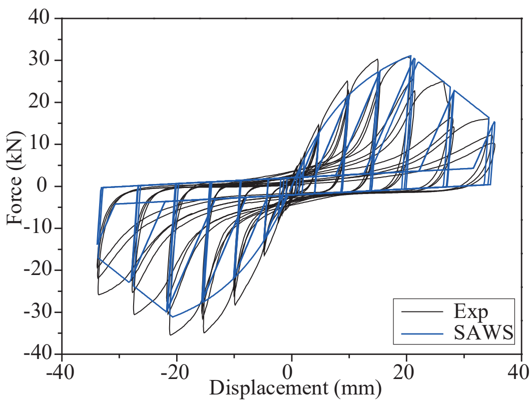

Figure 4: *Comparison of the load-slip curves of the angle brackets in shear*

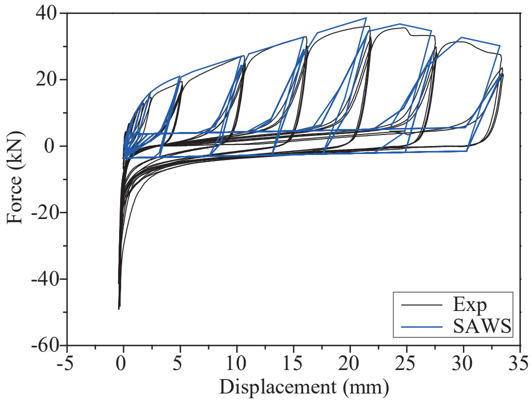

Figure 5: *Comparison of the load-slip curves of the angle brackets in tension*

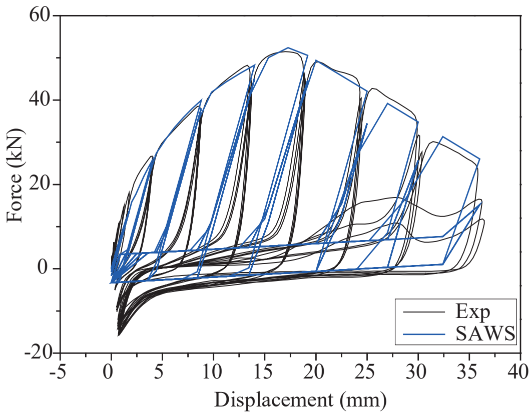

Figure 6: *Comparison of the load-slip curves of the hold-downs in tension*

Table 1: *Comparison of experimental and numerical results for component level modelling*

	Parameter	Exp	SAWS	Δ (%)
AB- shear	E_d (kNmm)	5759	5532	3.9
	F_{max} (kN)	30.8	31.1	1
AB- tension	E_d (kNmm)	2834	2990	5.5
	F_{max} (kN)	36.1	38.6	6.9
HD- tension	E_d (kNmm)	3784	3141	17
	F_{max} (kN)	51.5	52.4	1.7

where, E_d refers to the total energy dissipated and F_{max} is the maximum load.

4 MODELLING OF CLT WALLS

4.1 MATERIAL DEFINITION

Owing to the fact that cross-laminated timber panels are composed of layers of boards with alternating grain direction, the calculation of the mechanical properties is more complex than in wood. One of the most common approaches adopted to define the material properties of CLT is the Homogenised, Orthotropic plane stress Blass reduced cross Section (HOBS) method that approximates the multi-layer section as a single layer using pre-defined coefficients given in [14]. In this paper, however, orthotropic properties were assigned to the shell elements that composed the CLT wall panel models. Since the boards belonged to the C24 class, the base material properties were taken from [15]. The equivalent moduli of elasticity in the X and Y directions (refer Figure 7 for panel orientation) were calculated on a weighted average basis as 3407 MPa and 7963 MPa respectively. The equivalent modulus in the Z direction was set to 300MPa as suggested by [16]. The shear moduli were calculated in accordance with the equations prescribed by [17]. Since the panels were not glued on their narrow face, Poisson's ratio was taken to be 0. The value of the density of the panels was assumed to be 420 kg/m^3. The above properties were then assigned to ShellMITC4 elements that composed the CLT wall panel model.

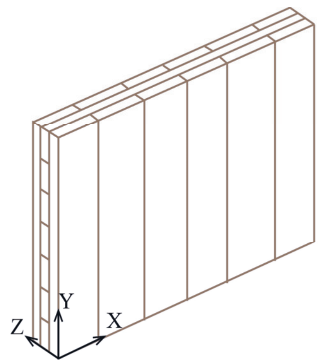

Figure 7: *Orientation of the CLT wall panel*

4.2 CONNECTOR DEFINITION

Although angle brackets are primarily designed as shear resisting elements, based on the experimental test results, it can be concluded that their contribution to the strength and stiffness in the axial direction cannot be ignored. Therefore, the angle brackets were considered to be bidirectional, having axial and shear behaviour, while the hold-downs were considered to be unidirectional with only axial capacity, using the hysteresis parameters obtained in section 3. The connectors were defined as zero length elements joining two coincident nodes, one end of which was fixed and the other which was attached to the base of the wall panel. In the single joint models as the loading consists of only half cycles, there is no need to define the high stiffness of these connectors in compression. However, in the case of the wall models, the definition of contact in compression is extremely important. With this in view, the behaviour of the springs in the axial direction was defined to follow a load path defined by the SAWS model in tension and to have contact behaviour defined by an elastic- perfectly plastic gap when subjected to compression.

4.3 WALL MODEL

The CLT wall models were meshed into 12 elements along their length and height, but taking care to respect connector positions in the experimental tests. In order to determine if a finer mesh would result in improved accuracy, separate wall models having mesh sizes of 26x26 were analysed. Since the analysis of these models was time-consuming and yielded the same results as the corresponding wall models meshed with 12x12 elements, the choice of using the model with lesser elements was made. It is inferred that the refinement of the mesh size does not seem to have any effect on the hysteresis behaviour of the walls, as long as there are no openings and the connector positions are not compromised. This is in line with the observations made during the experimental tests, where almost all of the deformation was limited to the connectors and the panel itself deformed only slightly.

The displacement time history was applied at the top left wall corner and the displacement at the midpoint of the top of the wall model was monitored, exactly as in the experimental set-up. The comparison of the experimental and numerical results for lateral force versus displacement curves for walls of Type A and B with different levels of vertical loading are plotted from Figure 8 to Figure 11. To determine the amount of energy dissipated by the models in comparison with the experimental results for the same, plots of energy dissipated in relation to the cumulative displacement are shown in Figure 12 to Figure 15.

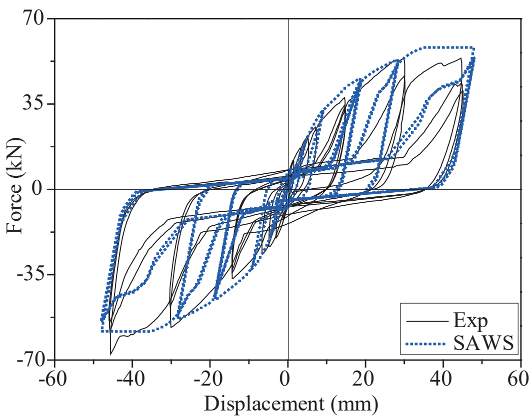

Figure 8: *Comparison of the load-slip curves of wall type A with no vertical loading*

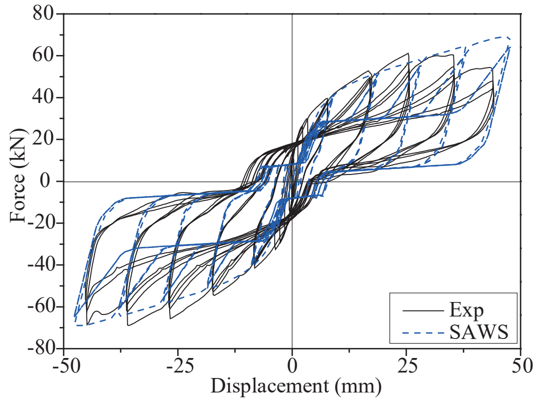

Figure 9: *Comparison of the load-slip curves of wall type A with a vertical loading of 20.8kN/m*

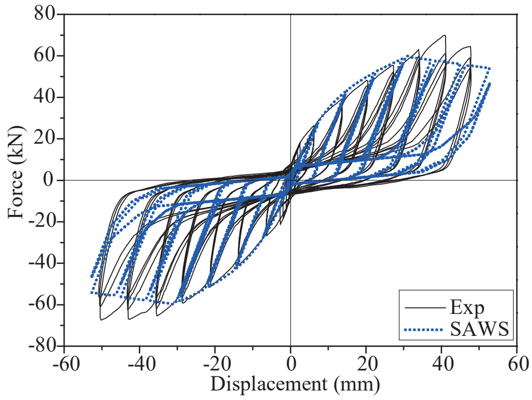

Figure 10: *Comparison of the load-slip curves of wall type B with a vertical loading of 5kN/m*

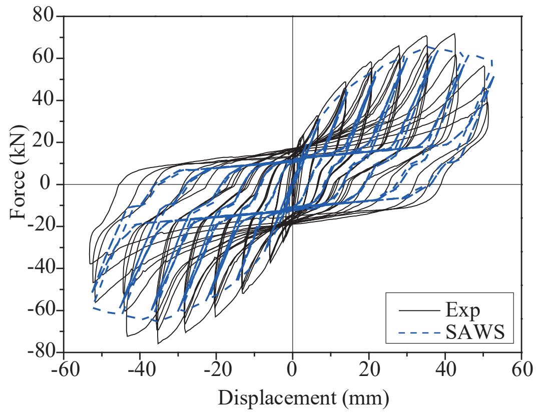

Figure 11: *Comparison of the load-slip curves of wall type B with a vertical loading of 20.8kN/m*

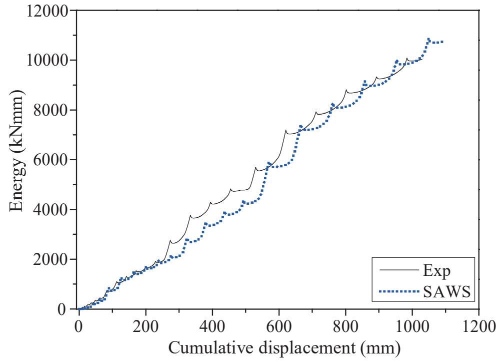

Figure 12: *Comparison of the energy dissipated by wall type A with no vertical loading*

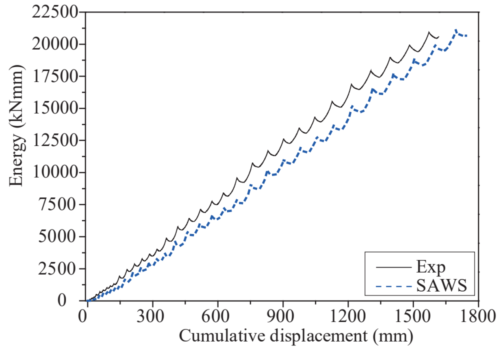

Figure 13: *Comparison of the energy dissipated by wall type A with a vertical loading of 20.8kN/m*

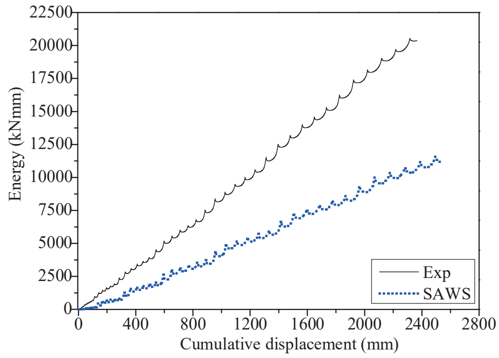

Figure 14: *Comparison of the energy dissipated by wall type B with a vertical loading of 5kN/m*

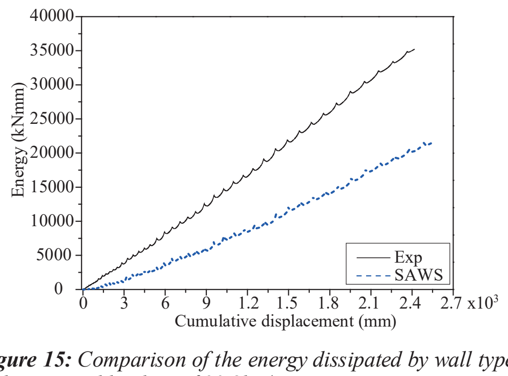

Figure 15: *Comparison of the energy dissipated by wall type B with a vertical loading of 20.8kN/m*

In the experimental tests, the primary source of deflection was due to rocking in all the four wall specimens. This was also noticed in all the models created in OpenSees. Walls of type A had a greater resistance to sliding than walls of type B because of the low shear resistance of the hold-downs. The amount of rocking also decreased with the increase in the magnitude of vertical load. These phenomena were evident even in the models.

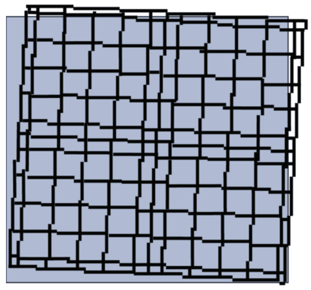

Figure 16: *Rocking of wall models*

The mechanical properties of the tested walls and their models were then calculated using the first envelope curves formed by joining the points of maximum load. The comparison of the mechanical properties of the wall specimens with the corresponding values yielded by the finite element model is made in Table 2. The properties taken into consideration were the initial stiffness (K_{ser}), maximum load (F_{max}), ductility (μ) and energy dissipation (E_d). The values for K_{ser} and μ were calculated based on the guidelines given in [18].

Table 2: *Comparison between experimental and numerical values of mechanical properties of different walls*

Wall		K_{ser} (kN/mm)	F_{max} (kN)	μ	E_d (kNmm)
A-0	Exp	3.41	53.78	3	10037
	SAWS	4.01	58.2	5	10768
A-20.8	Exp	9.75	61.18	13	20590
	SAWS	5.45	68.81	7	20679
B-5	Exp	2.56	69.98	2	20371
	SAWS	3.91	61.14	4	11179
B-20.8	Exp	4.69	71.73	4	35207
	SAWS	4.45	66.76	4	21422

The models of walls of type A overestimated the lateral load capacity while the models of type B walls yielded lower values of maximum load compared to their experimental counterparts. The walls with vertical loading of higher magnitude dissipated more energy than their counterparts with no load or lower magnitude of vertical load. The best agreement in energy dissipation values was obtained for wall specimen A2, where the OpenSees model dissipated 0.4% more energy than the walls that were experimentally tested. For wall A1, the OpenSees model overestimated the energy dissipated by just 7.3%. In the case of the type B wall models, the energy dissipated was much lower than in the experiments. In general, the agreement in the energy dissipation for walls of type A is much better than for walls of type B. This could be due to the fact that the shear component of the hold-downs was not modelled. Although the strength and stiffness of the hold-downs in shear is very low, the hysteresis loops are quite wide [19]. For the SAWS model as far as ductility and initial stiffness values are considered, the models were able to give good estimates of the experimental values, except in the case of wall A-20.8. Even though the ductility and initial stiffness values obtained from the load-slip curves of the model were higher than for the other models, they were much lower than the experimentally obtained values. In general, better values for initial stiffness may be expected if the influence of friction is also considered. A panel only starts sliding once the resistance to friction has been overcome which finally increases the stiffness parameter. Furthermore, the chosen method for determining the yield displacement, necessary for calculating the ductility value, is highly influenced by the initial stiffness; compare [18]. As a consequence, a slight difference in initial curve shape may lead to quite high differences in absolute ductility values; see also [8].

5 SENSITIVITY ANALYSIS

5.1 VERTICAL LOAD

In this sub-section, the influence of vertical loading on the behaviour of the CLT wall panels under lateral loading was further investigated. In CLT structures, the walls in the ground floor have to be equipped to deal with the weight from the storeys above them. To check the influence of vertical load (q) on the CLT wall panels used in the present study, 4 loading cases for each wall type were considered – 0kN/m, 5kN/m, 15kN/m and 20 kN/m. It must be noted that the cyclic displacement input for wall models within each type was the same. The results of the analysis for walls of type A and B have been tabulated in Table 3 and Table 4 respectively. In the case of walls having only angle brackets, the wall model with the vertical loading of 20 kN/m, the initial stiffness, maximum load and energy dissipation showed an increase of 36.6%, 17.8% and 29.8% respectively over the case of the model with no vertical loading. For models of wall type B, the model with the vertical loading of 20 kN/m showed a 24.3% increase in initial stiffness and a 14.8% increase in peak load, while the energy dissipation was almost double. In Figure 17 and Figure 18, it is clearly seen that with the increase in vertical loading, there was a change in the shape of the hysteresis loops near the origin, a phenomenon that was also reported by [20] based on their experimental campaign.

Table 3: *Comparison of mechanical properties of wall type A with different levels of vertical load*

q (kN)	K_{ser} (kN/mm)	F_{max} (kN)	E_d (kNmm)
0	3.99	58.17	15880
5	4.38	62.30	17311
15	5.18	65.93	18481
20	5.45	68.52	20607

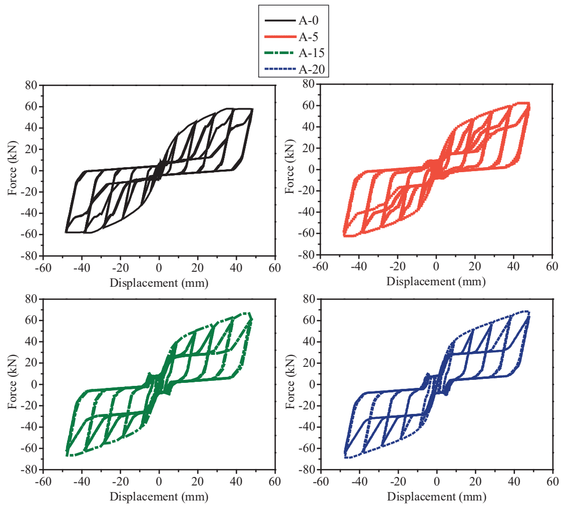

Figure 17: *Load-slip curves of type A wall models with different magnitudes of vertical loading*

Table 4: *Comparison of mechanical properties of wall type B with different levels of vertical load*

q (kN)	K_{ser} (kN/mm)	F_{max} (kN)	E_d (kNmm)
0	3.54	57.82	10390
5	3.91	61.14	11179
15	4.00	65.04	13329
20	4.40	66.35	20638

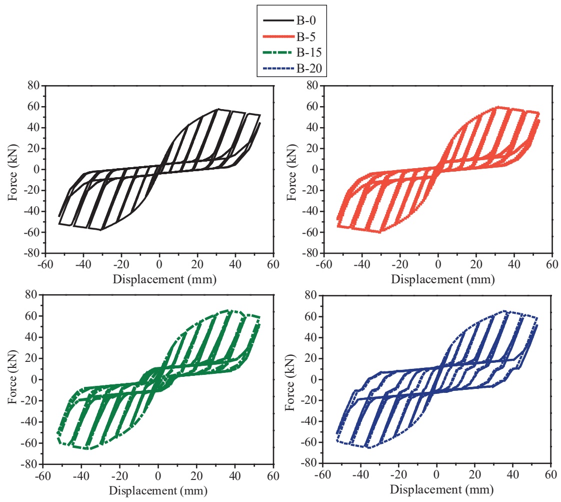

Figure 18: *Load-slip curves of type B wall models with different magnitudes of vertical loading*

It can be inferred that the increase in axial loading has a significant impact on the initial stiffness and the energy dissipation capacity. The proportion of the increase in the lateral load resistance was lower. According to [20], a vertical loading of at least 20 kN/m is required in order to have a significant influence on the lateral load resistance.

5.2 INFLUENCE OF OPENINGS

In order to determine the effect of the presence of openings in the CLT wall panel, quasi-static tests were conducted on a wall panel of the same dimensions as in the case of the walls without openings. Hold-downs were placed at the corners and angle brackets flanked the opening on either side. The panel with the dimensions of the opening is seen in Figure 19. The opening forms about 30% of the panel area. In one of the experimental campaigns conducted by [21], the effect of the presence of doors and windows in CLT panels was studied. The findings of this study revealed that openings with up to 30% of the panel area were found to have no effect on the load-bearing capacity of the wall panels. The openings, however, had an influence on the wall stiffness.

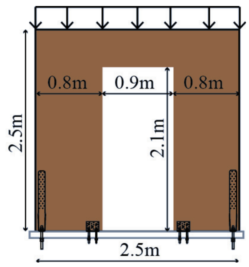

Figure 19: *Dimensions of wall specimen with openings*

In the case of the model of the wall panel with the opening, the initial stiffness was found to be 2.21 kN/mm and the maximum load attained was 72.59kN. The initial stiffness of the tested wall was 3.81 kN/mm while the maximum load attained was 75kN. So the model provided a good estimation of the lateral load capacity. In the interest of determining the effect of the opening on the lateral load resistance, the load-slip curves of this model were compared with those obtained from the model of wall B-20.8, which had a similar connection configuration and the same magnitude of vertical loading, but no opening (see Figure 20). The only difference was in the positioning of the angle brackets, which had to be shifted by 20.5cm on either side due to the presence of the opening.

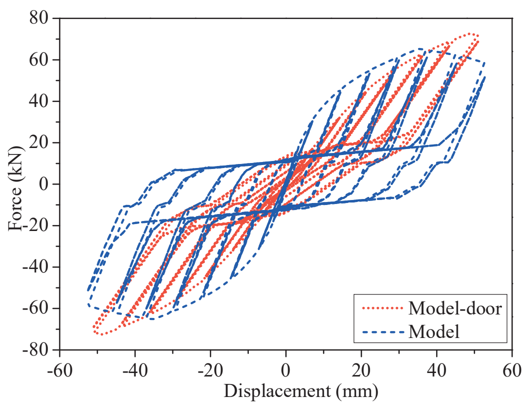

Figure 20: *Load-slip curves of wall panels with and without openings*

It is clearly seen that in the presence of an opening, there is a reduction in the initial stiffness. The presence of the opening also resulted in the hysteresis loops becoming narrower. The peak load attained by the model of the panel with the opening was about 8.7% higher than that of the model of wall B-20.8. This is because the angle brackets are shifted away from the centre, thus increasing the resistance to uplift and resulting in a higher resistance to lateral loads. This demonstrates that the position of the connectors has a strong influence on the lateral load resistance of the wall panel, which corroborates what was observed during the experimental tests. In the tests on the CLT wall panels, the failure of the angle brackets was found to be dependent on their position in the wall. When positioned closer to the corners, they underwent tension failure, while the angle brackets in the interior of the panel failed in shear [7].

6 CONCLUSIONS

The feasibility of using a suitable hysteresis model in OpenSees, an open access software, to replicate the force-deformation behaviour of CLT structures under cyclic loading at the component level has been explored in this paper. Since it has parameters to control the load-slip path and define the strength and stiffness degradation, the SAWS model was selected from the available hysteresis models.

Based on the results of the numerical analyses and corroborated by prior experimental tests, the inferences made have been presented herein. The energy dissipation by wall models where only angle brackets were used, was very similar to their experimental counterparts. The models of the walls with hold-downs and angle brackets as connectors yielded much lower values of energy dissipation than the values obtained in the experimental tests, which was perhaps due to the absence of a shear component for the hold-downs. An increase in vertical loading was found to significantly increase the initial stiffness and energy dissipation of the wall models, an effect which was also observed in the experiments. However, the increase in the lateral load resistance was not as high. The orthotropic material properties of CLT calculated as specified in the paper, were found to be suitable. The presence of openings in CLT panels reduces the initial stiffness of the walls, a phenomenon which was clearly exhibited even in the numerical model. This paper also demonstrated the impact of the position of connectors on the load carrying capacity of CLT wall panels.

Incorporating the effect of friction could have led to an improvement in the load-slip behaviour of the model. Nevertheless, this model can be used to simulate the behaviour of connections and to predict the lateral load capacity of CLT wall panels under cyclic loading with reasonable accuracy.

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